CHARACTERISTICS OF FLOW OVER WEIRS OF FINITE CREST WIDTH

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Manoj Kumar Shukla

CERTIFICATE

Certified that this work, "Characteristics of Flow over Weirs of Finite Crest Width" by Manoj Kumar Shukla, has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

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ABSTRACT

The results of an experimental investigation on two-dimensional free flow over rectangular weirs with a sharp square entrance as well as streamlined entrance, horizontal crest of finite width, and vertical faces are presented in the thesis. The discharge coefficient and the flow pattern have been studied for different types of flow over these weirs.

The discharge coefficient has been found to be primarily a function of the ratio of the head H over the weir to the width B of the weir and the ratio of the head over the weir to the height P of the weir. When the ratio of the head over the weir to the width of the weir is greater than or equal to 1.8 the weir behaves as a sharp-crested weir. The discharge coefficient for weirs of finite crest width with a streamlined upstream corner has been found to be independent of H/P for H/B up to about 0.22 and H/P up to about 1.0 and the well-established relationship between the end depth and the critical depth ($Y_b/Y_c = 0.715$) for horizontal channel has been found to be applicable.

The discharge computation can be made by a single measurement of the end depth with the help of the graphs presented in this thesis for the weirs of finite crest width with a sharp upstream corner as well as streamlined upstream corner.

CHAPTER 1

INTRODUCTION

The problem of flow over weirs has engaged the attention of the most eminent hydraulicians of the past few centuries. Considerable amount of work has been done on this basic contrivance of flow measurement but still much more remains to be done especially on the flow over weirs of finite crest width.

The discharge coefficient of such a weir depends upon the geometry of the structure, geometry of the approach channel, characteristics of the flow, and the properties of the fluid.

The most important point in all projects which involve use of water is to assess the discharge most economically. For this various types of measuring devices are used. Overflow and underflow structures represent the most common means of producing a channel control and a great deal of information in particular design is available in the literature. The most economical method of flow measurement will be that wherein an existing structure is used as a measuring device. In most of the hydraulic projects, there will be weirs of varying dimensions which maintain a unique relation between the stage and the discharge and as such can be used as control devices. Weirs are particularly useful gauging devices in case a predetermined rate of discharge is to be maintained as it is required only to fix the stage of water above the weir crest. Because of their simplicity and

accuracy over a wide range of discharges, weir gauging is preferred amongst other methods for laboratory work also.

The purpose of the present investigation is to make available the basic information, sufficiently comprehensive and precise, regarding the discharge characteristics of weirs of finite crest width which is going to be helpful in the design of hydraulic structures. The characteristics of flow over a simple rectangular weir with a horizontal crest of finite width, with an upstream sharp corner as well as streamlined upstream corner, vertical faces, which occupy the full width of the flume, have been investigated.

Flow pattern over weirs of finite crest width is complicated being predominantly curvelinear besides consisting of regions of eddying, accelerating, and retarding fluid masses. Because of the flow pattern and the combined influences of the properties and geometric variables, the discharge function is not yet subjected to rigorous mathematical analysis. The most direct method which is usually adopted in such cases is to depend on experimental results.

The flow over weirs may be in the modular or in the nonmodular ranges. In the former, the discharge is uniquely controlled by the upstream depth of flow and in the latter, the discharge is a function of both the upstream and downstream

depth of flow. The stage at which the change occurs is termed as modularity limit. The present investigation was carried out in the modular range.

CHAPTER 2

THEORY AND REVIEW OF LITERATURE

2.1 Types of Weirs:

There are several types of weirs in practice, each type being distinguished from the other by the shape of the weir and the flow profile. Depending upon the shape of the weir they can be classified as follows:

1. Rectangular weirs, and
2. Embankment shaped weirs

Depending upon the flow profile Muralidhar (19) has classified them into four categories.

1. Sharp-Crested Weirs
2. Narrow-Crested Weirs
3. Broad-Crested Weirs
4. Long-Crested Weirs

Sharp-crested weir has very small crest width followed by a suitable chamfering and is expressly constructed for flow measurement. If H is head of water over the weir and B is the crest width, Muralidhar reported that for values of H/B greater than 1.5 the lower nappe springs clear off the crest touching the weir only at the upstream corner and the weirs with such a flow profile are classified as sharp-crested weirs. It was

noticed that, for a narrow range of H/B greater than 1.5 the conditions were quite unstable with lower nappe intermittently springing clear off the crest and reattaching to the crest. The stage at which the change takes place depends upon the conditions of flow.

A weir was defined as a narrow-crested weir if the value of H/B is greater than 0.4 and less than 1.5. For the values of H/B between 0.1 and 0.4 it was defined as a broad-crested weir and when H/B is less than 0.10 and greater than 0.03 it was reported as a long-crested weir. Generally, the last three categories are popularly known as broad-crested weirs.

A broad-crested weir, as it is popularly known, is a weir of finite crest width larger than that of a sharp-crested weir. These weirs generally form part of hydraulic structures built to serve several purposes in the irrigation and soil conservation works. These can be easily built in the field and are more suitable. There are again two different categories of broad-crested weirs as follows:

1. Broad-crested weirs with a sharp upstream corner
2. Broad-crested weirs with streamlined upstream corner

Full width rectangular weirs with sharp square entranc horizontal crest of finite width and vertical faces fall into the first category and those with streamlined upstream corner fall into the second category.

2.2 Sharp-Crested Weirs:

The flow over a rectangular sharp-crested weir in the freely discharging conditions has been very extensively investigated since Polini (15). Polini was the first to publish an equation for weir discharge as follows:

$$q = 2/3\ C_c \sqrt{2g}\ H^{3/2}$$

wherein q = Discharge per unit length of the channel

H = Head of water over the weir

C_c = Coefficient of contraction

But he did not take into account the nappe contraction which was later taken into consideration by Du Buat (15). Weisbach applied the velocity of approach correction and gave the discharge equation as follows (15):

$$q = \frac{2}{3} C_c \sqrt{2g} \quad (H + \frac{V_0^2}{2g})^{3/2} - (\frac{V_0^2}{2g})^{3/2}$$

wherein V_0 = Velocity of approach and

H = Head of water over the weir

Several empirical formulae have been advanced, each characterising the individual experimentation (11, 25, 12). Kindsvater and Carter presented a comprehensive analysis for flow over rectangular thin plate weirs based on existing literature and tests at the Georgia Institute of Technology (15).

Rehbock gave the following equation for the sharp-crested weirs (5):

$$q = \frac{2}{3} C_d \sqrt{2g}\, H^{3/2}$$

wherein

$$C_d = 0.605 + 0.08\, H/P + \frac{1}{1000\, H}$$

in which P represents height of the weir

First term in the above expression for C_d represents basic contraction coefficient, second term represents combined velocity of approach influences and last term of the equation is due to Weber and Reynolds number influences. There was

a controversy raised on this subject by Engel and Stainsby (5) who supported the theory of M. Rethy and Von Mises and other authorities who presented on exact mathematical lines the variation of the contraction coefficients over the complete range of area ratios from 0 to 1.

2.3 Weirs of Finite Crest Width with a Sharp Upstream Corner

Characteristics of flow over square edged weirs of finite crest width with a sharp square entrance, and vertical faces occupying the full width of rectangular channel have been investigated by several investigators (11, 14, 25, 29) since Blockwell (~~1852~~). Several formulae, each designed to fit in a particular set of experimental data, have been proposed. There is lot of disagreement in the literature regarding the discharge characteristics of these weirs. An analysis of the available literature has been presented by Tracy (29), Engel and Stainsby (14) and Muralidhar (19). Certain aspects of the flow pattern, velocity distribution and pressure distribution have been presented by Rouse (21), Kikkawa, Ashida and Tsuchiya (14), Kindsvater (16), Woodburn (30) and Doeringsfeld and Barker (1).

An empirical discharge coefficient is used in the discharge equation to cover the various aspects of fluid dynamic The common flow equation is derived from first principles, the Bernoulli equation, referring to nonviscous and irrotational flow conditions. All attempts have failed to establish the

contraction coefficient on strict mathematical reasoning. Few investigators have attempted the problem mathematically by using conformal mapping or Boundary Layer Theory (7). Common types of weir equations are valid over limited ranges and are related to some specific features of their particular investigation.

The views of various investigators regarding the variation of the discharge coefficient with parameters involved in the phenomenon have been contradictory, particularly regarding the effect of the ratio of head over the weir to the height of the weir (18). Besides, the data given by various investigators covers a limited range of the variables involved. Muralidhar (19) conducted a detailed study on the discharge characteristics of weirs of finite crest width and found that the discharge coefficient is some function of H/B where B is the crest width of the weir and H is head over the weir. He reported that the discharge coefficient is independent of H/P, where P is the height of the weir. It is to be noted that for his experiments H/P ranged from 0.1 to 0.9, which is too short a range. Hence, a detailed study of the discharge characteristics of rectangular weirs of finite crest width with a horizontal crest, square entrance and vertical faces was undertaken to cover higher ranges of the variables involved.

End Depth for Weirs:

It was stated by Rouse (21) and Keutner, based on their studies on flow profile over weirs of finite crest width, that

end depth varies with the width of the weir. He also reported a constant ratio of the end depth to the hydrostatic critical depth in flow over rectangular broad-crested weirs. Kindsvater contributed towards this line by reporting that the depth of flow obtaining at the end of the weir is greater for weirs of large crest width.

Despite the fact that the hydrostatic critical section for parallel flow has no definite location, but changes position with discharge, channel slope, and roughness, it is evident that brink section in a free overfall is a true section of minimum energy, and as such is a suitable control point at which a single measurement of depth should permit a simple and direct determination of the discharge.

In flow over sharp-crested weirs the nappe springs clear off the crest touching it at theupstream crest corner. It is known that critical depth occurs at a section where lower nappe reaches its maximum elevation (19). Thus the ratio of the end depth to the hydrostatic critical depth is greater than unity. As the width is increased flow characteristics are modified for narrow-, broad- and long-crested weirs, thus the end depth which is greater than hydrostatic critical depth reduces to a minimum value and approaches a constant value of 0.715 given by Rouse (21) for channel range. Rouse (21) took the data of Prentice who conducted experiments on weirs with square or rounded entrance and crest width of 1, 2, and 3 feet and found that the ratio of the end depth to the hydrostatic

critical depth for horizontal rectangular free overfall is 0.715.

Tracy (29) and others indicated that the end depth for weirs with a square entrance is lower than that for weirs with rounded entrance. Many other investigators including Rajaratnam and Kindsvater observed that the end depth ratio was not constant(9)

The results relating to the end depth for full width rectangular weirs with confined nappe are also presented in this thesis.

2.4 Weirs of Finite Crest Width with a Streamlined Upstream Corner

It is established that a section of critical depth on the weir exists, if it is broad-crested, but the location of the section and whether it is in a zone of approximately hydrostatic pressure distribution, depends primarily upon the form of the weir profile.

If the upstream corner of the weir is not streamlined separation will occur and the control section will coincide with the maximum elevation of the separation surface, where the specific head attains its true minimum value. If on the other hand upstream corner is well streamlined the section of minimum specific head will be located downstream (22). Few investigators have attempted to study the effect of various parameters involved in the determination of the discharge coefficient(10)

The broad-crested weir can be considered a sufficiently precise control for metering purposes only if properly calibrated. This is due, in large measure, to the fact that variation in boundary layer growth with weir size, form and discharge causes the line of total head to change accordingly in slope. This effect is calculated by boundary layer theory by Hall (7).

For broad-crested weirs, the customary discharge equation is $Q = C\ L\ Y_c\ V_c = 3.09\ C\ L\ H^{3/2}$, where Y_c is critical depth in feet, V_c is critical velocity in feet/sec, and L is length of the weir in feet which is same as width of the flume, and C is coefficient.

Rouse (22) gave values of C for various values of H/B, ranging from 0.05 to 0.15, calculated for weirs of various crest width ranging from one foot to 50 feet and for constant value of L/B equal to 0.20. These values of C indicate the so called scale effect (the relatively better performance of large structures). In the latter part of the thesis the results relating to the free flow over rectangular weirs of finite crest width with a horizontal crest, streamlined entrance and vertical faces, have been presented. This was carried out to find whether the coefficient of discharge is dependent on H/P or not and to verify the values of discharge coefficient given by Rouse. A limiting value of H/B for which a true critical-flow section exists on the weir crest for broad-crested weirs with streamlined upstream corner was also found.

CHAPTER 3

ANALYSIS OF THE PROBLEM

The general flow pattern, pressure, and velocity distribution for flow over weirs of finite crest width were studied by several investigators (14, 16, 29, 21, 30). The flow pattern being very complex, is not yet subject to a complete theoretical analysis. A rational formula was developed by Unwin and Frizell (11) by assuming rectilinear discharge of an inviscid fluid over a weir of sufficiently long crest to develop parallel flow within the crest. The formula suffers from several descrepancies (23). The formula was corrected by some (7, 16) to take into effect the growth of the boundary layer. Bazin also investigated flow over weirs of finite crest width under different conditions of nappe.

The broad-crested weir is an intermediate case between the one in which the flow is wholly curvelinear and the one in which boundary resistance predominates, and hence both the accelerative and viscous effects must be considered in its analysis. The upstream end of such a weir corresponds to a channel inlet and downstream end to a channel outlet, with central reach which is sufficiently long for an appreciable boundary layer to develop and yet not long enough to be wholly free from curvelinear effects of the two end zones (22). Under these conditions, it is impossible to attempt any mathematical approach.

Fig. 1 gives the definition sketch of flow over a full width rectangular weir with a sharp square entrance, horizontal crest of finite width and vertical faces. The discharge Q is given by:

$$Q = \frac{2}{3} C_d \sqrt{2g}\, L \left\{ \left(H+\frac{V_0^2}{2g}\right)^{3/2} - \left(\frac{V_0^2}{2g}\right)^{3/2} \right\} \qquad (3.1)$$

where C_d = Coefficient of discharge

V_0 = Velocity of approach

H = Head over the weir

L = Length of the weir, which is same as the width of the channel

A detailed discussion of the several modification to the above formula has been given and finally the following form was proposed by Horton (11):

$$Q = C\, L \left(H + \frac{V_0^2}{2g} \right)^{3/2} = C\, L\, H_0^{3/2} \qquad (3.2)$$

where $C = \frac{2}{3} C_d \sqrt{2g}$

H_0 = average energy head referred to crest level

Few investigators considered the influence of velocity of approach to be taken care of by the discharge coefficient C_d and adopted the following form:

$$Q = \frac{2}{3} C_d \sqrt{2g}\; L\, H^{3/2} \qquad (3.3)$$

where H is head over the weir.

This form is used in the present investigation.

Dimensional Analysis:

The variables describing the flow of water over a weir of finite crest width shown in Fig. 1 are the height of the weir (P), the width of the weir (B), the length of the weir (L), the discharge (Q), the head over the weir (H), the average energy head (H_0), the specific weight of the fluid (γ), the density of the fluid (ρ), the dynamic viscosity (μ), and surface tension (σ) and height of the roughness k. The variables describing the characteristics of flow over these weirs are expressed in a fundamental relationship as

$$f_1 \ (H, \ Q, \ P, \ B, \ L, \gamma, \rho, \mu, \sigma, k) = 0 \qquad (3.4)$$

Dimensional analysis loads to the following relationship:

$$C_d = f_2 \ (H/P, \ H/L, \ H/B, \ R, \ W, \ k/H) \qquad (3.5)$$

wherein R is Reynolds number and W is Weber number.

The exact functional relationship is not known.

A detailed discussion on the significance of these dimensionless parameters was presented by Tracy (29), Kindsvater and Carter (15).

As the data available on the discharge characteristics of weirs of finite crest width is limited, besides the contradicting conclusions, the ingenious method of accounting the

effects of viscosity and surface tension put forth by Kindsvater and Carter could not be attempted here.

In the present investigation heads above a minimum of one inch were only considered. Also width and height of the weirs were kept greater than the suggested minimum values. The effects of variation of the Reynolds Number, and the Weber Number, and relative roughness have not been studied. Their variation in the experiment is not much. Thus the discharge coefficient is given by

$$C_d = f\ (H/P,\ H/B,\ H/L) \qquad (3.6)$$

The effect of the parameter H/P, representing the area contraction, was considered by several investigators. Singer (27) presented a simple curve for the variation of the discharge coefficient with H/B for $\frac{H}{H+P} = 0.30$ and a series of curves for values of $\frac{H}{H+P}$ in the range of 0.30 to 0.60. Some investigators (19) concluded that the discharge coefficient is independent of H/P. Kindsvater's (16) elaborate experiments on "Discharge Characteristics of Embankment Shaped Weirs" showed that the coefficient is virtually independent of H/P. It may be noted that his data includes values of H/P ranging from about 0.1 to 2.4 (values of H/H+P in the range 0.08 to 1.8). The parameter H/L, representing the aspect ratio of the channel, was concluded to be insignificant by several investigators. Muralidhar (19) plotted the data, for all the series of experiments he conducted, with H/B on x axis and C on Y axis, along with other parameters H/P and H/L. It was shown that

points with a wide range of values of H/P and H/L coincided and defined that C is a function of only H/B. Thus in the range of his investigations, C was independent of H/P, as concluded by some other investigators.also. He accepts that the range for H/P was very limited i.e. between 0.1 to 0.9, which is not sufficient to say with certainty whether discharge coefficient is really independent of H/P or not. The parameter H/L may come into effect for very narrow channels. The effect of H/P at values higher than covered in Muralidhar's work and previous investigations is considered in the present investigation. In the present investigation the following equation has been evaluated on the basis of systematic experimental investigations:

$$C_d = f\ (H/P,\ H/B) \qquad (3.7)$$

Same procedure was adopted for studying the coefficient of discharge for streamlined weirs of finite crest width also. Equation 3.3 was used for calculation of discharge coefficient. Rouse (21) used the equation $Q = 3.09\ C\ L\ H^{3/2}$ for the evaluation of C. For the purpose of comparison, the values of C_d were converted into C by the following equation:

$$C = \frac{2}{3}\ \frac{C_d\sqrt{2g}}{3.09} \qquad (3.8)$$

<u>End Depth Analysis</u>

In both the cases, i.e. square-edged and streamlined weirs of finite crest width, discharge was measured by

calibrated rectangular notch. Critical depth was calculated from the following equation:

$$\frac{Q^2}{g} = \frac{A_c^3}{T} \qquad (3.9)$$

where A_c = Area of the critical flow section

T = Width of water surface

In particular, Y_c for flow in rectangular section can be calculated from the following equation:

$$Y_c^3 = \frac{q^2}{g} \qquad (3.10)$$

The ratio of the end depth to the hydrostatic critical depth was calculated for few tests on flow over full width rectangular weirs of finite crest width. Knowing the end depth, it is possible to calculate discharge from the following equation:

$$Q = k \cdot L \sqrt{g} \; Y_b^{3/2} \qquad (3.11)$$

where Y_b = brink depth, popularly known as end depth and

k = A coefficient.

From the above equations, one can get the relation

$$k = \left(\frac{Y_c}{Y_b} \right)^{3/2} \qquad (3.12)$$

Rouse (21) reported the value of Y_b/Y_c as 0.715 for broad-crested weir and so k for such case is $(\frac{1}{0.715})^{1.5} = 1.654$. In the prese investigation, the effect of H/B and H/P on k for broad-crested weirs has been evaluated.

To summarise, in the present thesis, the following equations have been evaluated on the basis of experimental investigations

$$C_d = f\ (H/B,\ H/P) \tag{3.7}$$

$$Y_b/Y_c = f\ (H/B,\ H/P) \tag{3.13}$$

$$k = f\ (H/B,\ H/P) \tag{3.14}$$

CHAPTER 4

EXPERIMENTAL DETAILS

4.1 Experimental Equipment:

Experimental equipment used in the present investigation is shown in Fig. 2.

Water was supplied through an 8" supply line from a constant head tank and it was possible to obtain a maximum discharge of 4.5 cusecs. Water was discharged into the head tank of the flume, 3 feet square and 5 feet 8 inches deep. The tank was provided with a vertical-lift type of control gate, which could be moved by a rack and pinion mechanism. Water from the head tank entered the horizontal flume 15' 3" long, 2' 0" deep, and 1' 6" wide. This flume had its floor made of steel plates and side walls made of glass plates. A streamlined transition was provided between the head tank and the flume.

Two wooden baffles were provided in the head tank to break the large eddies and to provide a flow which is homogeneous across the width of the flume. Besides, in certain cases, wooden baffles were provided at the water surface and the sufficiently large length of the flume ensured uniform approach flow for the weir.

Discharge was measured by letting the water flow into a drain, as shown in the figure, in which a contracted sharp-crested weir was fixed. The dimensions of this measuring weir

are shown in Fig. 4. The contracted sharp-crested weir was calibrated volumetrically with the help of a calibration tank, 20' 0" x 10' 0" x 7' 0". The drain in which the notch was installed was 21" deep and 21" wide. A gauge well was constructed by the side of the drain at a distance 3 feet upstream from the notch, to measure the head over the weir. The water finally discharged into the underground tank from where it was pumped back to the constant head overhead tank. A point gauge of range 3 feet and least count 0.001 foot was used for the measurement of water surface elevation.

Test Models:

The test weirs were made out of aluminium plate. A typical test weir is shown in Fig. 3. Dimensions of the weir models were so selected to cover values of H/B = 0.08, 1.0, 1.6, 2.0, 2:5, 3.0, 3.5, 4:0, 6.0, and 10.0 and H/P for each value of H/B to vary uniformly from 0.10 to 7.0. This was achieved by the combinations of horizontal plates and Z sections of various dimensions. In case width of the weir is considerably large, intermediate wooden supports were provided. B, the width of the weir, varied as 1/4, 1/2, 1, 2, 3, 4, 5, 10, 20, 30, 40 and 50 inches and the height of the Z sections varied as 1, 2, 3, 4, 5 5, 6, 7, 8, 9, and 10 inches. With the help of 12 plates and 10 sets of Z sections, it was possible to obtain all desired values of H/B and H/P.

The Head, H, over the weir was kept between a minimum of one inch to a maximum of seven inches. In all cases the weir models were fixed at the end of the flume so that the lower nappe was automatically ventilated. The flow, however, was kept confined by continuing the walls of the flume downstream from the weir also.

Weir models for studies on the streamlined weirs of finite crest width were also made in the same fashion. Height and length of the weirs and the head over the weirs were so selected as to cover the range of H/P from 0.1 to 1.0 for each value of H/B. The values of H/B covered were 0.05, 0.10, 0.15, 0.18, 0.2, 0.22, 0.25, 0.3, 0.4, 0.5, and 0.6.

The major and minor axis for the elliptical upstream corner piece in each case were chosen on the basis of the profile of a 2-dimensional jet emerging out from a 2-dimensional slot of width b in a two dimensional conduit of width B as the flow over a weir may be considered to be similar to this if the axis of symmetry in the former case is considered similar to the upper nappe in the latter case (22). An elliptical corner piece with a semi-major axis of 9 inches and a semi-minor axis of 3 inches was found to be suitable for all cases to ensure flow with no separation.

4.2 Procedure:

The head over the weir was measured at a section upstream from the weir sufficiently beyond the region of surface

draw down and at the same time not very far off to include the energy loss in the distance between the gauging section and the weir. Same procedure was adopted for measuring the head over the rectangular notch. Zero readings were taken with the point gauge set exactly in the desired position, before and after the experiment. The water surface at the brink of the weir was rapidly fluctuating. This depth was measured by a point gauge fixed in position to be exactly at the brink section. Care was taken to see that the point gauge was set properly as any slight displacement would result in serious errors.

The following procedure was adopted in all the tests. Water was let into the head tank by operating the sluice valve in the supply line. Sufficient time was allowed for the water to become steady and this was checked by repeated observations of the elevation of the water surface at the weir. Then the reading of the head gauge, gauge at the brink of the weir, and the gauge at the measuring weir were noted. Water surface elevation at short intervals in the horizontal direction was also noted to obtain full flow profile.

Calibration of Measuring Weir:

The rectangular contracted weir, installed in the drain for the measurement of discharge, was calibrated by letting the water flow over it and collecting it in a tank for known amount of time. For various heads over the weir, discharge was thus calculated. Kindsvater and Carter (15) gave an expression

for calculating discharge for thin-plate weirs.

$$Q = C_e b_e h_e^{3/2} \tag{4.1}$$

wherein C_e = Coefficient of discharge = f (b/B, h/P)

b = Contracted width of the weir

B_1 = Width of the channel

h = Head over the weir

P = Height of the weir crest

h_e = Effective head = h + 0.003 feet

b_e = Effective width = $b + k_b$, and

k_b = $f(b/B_1)$

In the present case b = 1.25 feet, $b/B_1 = 0.7$ and $k_b = 0.0135$ feet. Hence $b_e = 1.2635$ feet.

$$C_e = \frac{Q}{1.2635(h+0.003)^{3/2}} \tag{4.2}$$

C_e was calculated for various values of h/P and plotted with respect to h/P. By the method of least squares, the following equation was obtained:

$$C_e = 3.275 + 0.22\ h/P \tag{4.3}$$

This approach was adopted so that the results can be compared by the one obtained by Kindsvater. Kindsvater and Carter (15) obtained the equation for $b/B_1 = 0.70$ as

$$C_e = 3.18 + 0.154\ h/P \tag{4.4}$$

The difference may be due to different experimental conditions. Equation 4.3 was used for all the calculations.

CHAPTER 5

EVALUATION OF DATA AND DISCUSSION OF RESULTS

5.1 Accuracy Analysis:

From Equations 3.3 and 4.1 following expression for C_d is obtained:

$$C_d = \frac{3}{2} \frac{C_e \; b_e \; h_e^{3/2}}{\sqrt{2g} \; H^{3/2} \; L}$$

From the theory of errors, the expression for the percentage error in C_d can be obtained as follows:

$$\left| \frac{\Delta C_d}{C_d} \right| = \left| \frac{\Delta C_e}{C_e} \right| + \left| \frac{\Delta b_e}{b_e} \right| + \frac{3}{2} \left| \frac{\Delta h_e}{h_e} \right| + \frac{3}{2} \left| \frac{\Delta H}{H} \right| + \left| \frac{\Delta L}{L} \right|$$

From the above expression it is evident that the percentage error in C_d is mainly derived from the percentage error in the measurement of head. Percentage error in C_d as contributed by head measurements will be more for lower values of H and h i.e. head over the experimental weir and head over the measuring weir. If the maximum error in the measurement of head is considered as 0.001 feet and if the errors in C_e, b_e and L are considered negligible, if the data for head less than 0.08 feet are discarded the maximum percentage error would be 3.75% plus the percentage error in Ce.

5.2 Discharge Coefficient for Weirs of Finite Crest Width with a Sharp Upstream Corner

As the number of experiments were limited because of

lack of time, the variation of the discharge coefficient with H/P was studied for weirs with H/B = 0.08, 1.0, 1.6 and sharp-crested weirs.

The discharge coefficient was computed on the basis of Equation 3.3. The data for all the series are plotted with H/P on X axis and C_d on y axis along with H/B as the third parameter, as shown in Fig. 5. It can be seen that C_d is not independent of H/P as indicated by some investigators (19). It was also observed that the variation of the discharge coefficient with H/P follows a single trend up to H/P equal to about 6.0.

The discharge coefficient was found to vary with H/P as follows for different values of H/B:

Values of H/B	Equation for C_d	Limiting values of H/P for which the equation is valid
0.08	$C_d = 0.482 + 0.02\ H/P$	$\leqslant 6.0$
1.0	$C_d = 0.527 + 0.049\ H/P$	$\leqslant 6.0$
1.6	$C_d = 0.578 + 0.061\ H/P$	$\leqslant 6.0$
$\geqslant 1.8$	$C_d = 0.611 + 0.08\ H/P$	$\leqslant 6.0$

For various values of H/B $\geqslant$ 1.8 and H/P, discharge coefficient has been computed and plotted in Fig. 5. Values of the discharge coefficient, obtained experimentally, are compared with the

values of the discharge coefficient given by Rehbock's equation (1913):

$$C_d = 0.611 + 0.08\ H/P$$

and it is found that except for few points most of the points follow the above equation with maximum deviation of 3 to 4% from Rehbock's line. The discharge coefficient for the sharp-crested weir is a function of H/P only and is very sensitive to the crest conditions.

From Fig. 5, it may be seen that the lines for various values of H/B diverge as higher values of H/P are approached. Hence for lower range of H/P the coefficient was assumed to be independent of H/P (19).

5.3 Discharge Coefficient for Weirs of Finite Crest Width with a Streamlined Upstream Corner

The results relating to the characteristics of flow over full width rectangular weirs with a streamlined entrance, horizontal crest of finite width and vertical faces are presented in Tables 7 and 8 (Appendix – II)

The values of the discharge coefficients for various values of H/B ranging from 0.05 to 0.60 and H/P ranging from 0.1 to 1.0 are presented in Table 1, and these values are compared with the values reported by Rouse (22). Values of C are obtained by the following relationship:

$$C = \frac{2}{3} \cdot \frac{C_d \sqrt{2g}}{3.09}$$

It can be seen that except for the first value, other values are quite close to the theoretical values of C given by Rouse. Maximum percentage deviation is 4.38%. This much variation can be attributed to the fact that Rouse reported the values of C for L/B = 0.20 whereas in the present case L/B lies between 0.20 to 0.90. It should be noted that there is not much variation, worth appreciating, in the values of discharge coefficient within the following specified range of parameters:

$$0.05 \leqslant H/B \leqslant 0.25 \quad \text{and} \quad H/P < 1.0$$

Hence, within small range of H/P, discharge coefficient may be assumed to be independent of H/P.

5.4 Water Surface Profiles for Weirs of Finite Crest Width with a Sharp Upstream Corner:

1. Weirs with H/B = 0.08:

A series of waves were formed in the above range. Dimensionless flow profiles are shown in Fig. 8 for various values of H/P = 0.7866, 0.9916, 1.333, 1.9685 and 3.8057.

2. Weirs with H/B = 1.0:

The water surface was almost parallel to the crest, preceded and followed by the drawdown curves at the entrance and the exit of the weir. Dimensionless flow profiles for H/B = 1.0 and H/P = 0.5, 0.9498, 1.8182, 2.8037, 4.0 and 6.05 are shown in Fig. 7.

3. Weirs with $H/B = 1.6$

This is almost the limiting value of H/B, above which the weir behaved as a sharp-crested one. It was observed in various experiments that for values of $H/B = 1.8$ or greater (in some cases for values of H/B between 1.6 and 1.8), the surface profile was curvelinear and the lower nappe separated from the weir crest at the upstream corner and there was no reattachment on the weir crest. It was noticed that for a narrow range of H/B lower than 1.8 and greater than 1.6 the conditions were quite unstable with lower nappe intermittently springing clear off the crest and reattaching to the crest. The stage at which the change took place depended upon the conditions of flow. This has been defined very vaguely and hence an attempt was made by Muralidhar to present the approximate stage at which this change occurs based on the lower nappe profile data of Kandaswamy and Rouse for thin plate weirs (12). The horizontal distance at which the lower trajectory of the nappe, springing clear off the crest, would meet the level of the crest was taken as the limiting width of the weir for a given head. For weirs with crest width greater than this value, the nappe springing clear off the crest at the upstream corner would reattach to the weir crest. The water surface would be completely curvelinear.

Dimensionless flow profiles for $H/B = 1.6$ and for values of H/P = 0.6332, 1.066, 1.92, 3.12, 4.486 and 7.27 are presented in Fig. 6.

5.5 Water Surface Profiles for Weirs of Finite Crest Width with a Streamlined Upstream Corner

Water surface profiles for all cases were recorded and it was observed that if the upstream end ofthe weir is streamlined sufficiently, the roller and the accompanying rapid flow will no longer exist. If the upstream end of the weir is a sharp corner a region of discontinuity forms between flow and floor of the weir, the region being filled with a roller of water. Since the roller is not a part of live flow, the discharge passes over it as though the roller were a small spillway built upon the weir. Because of the effective height of this roller above the weir floor, the flow reaches at a velocity somewhat above the critical.

In the case of streamlined weirs of finite crest width the critical depth was found to occur when the transition region of crest is reached.

It was also noted that the position and number of surface undulation depend entirely upon the geometrical relation between head and weir dimensions. Any undulation for a given weir profile will move downstream with increasing discharge.

5.6 End Depth for Weirs of Finite Crest Width with a Sharp Upstream Corner

The ratio of the end depth to the critical depth Y_b/Y_c is constant for horizontal smooth channels and several investigators have confirmed this. Rouse (21) reported this value to be 0.715. He also reported that the end depth itself remains

unchanged in its relation to the computed critical depth. Values of Y_b/Y_c are plotted with respect to H/P and for values of H/B = 0.08, 1.0 and 1.6, and this is presented in Fig. 9. It can be seen that for lower values of H/P there is large variation in Y_b/Y_c and for values of H/P greater than 1.0 the variation is small. For H/B = 0.08 it is found that after H/P = 1.5 the value of Y_b/Y_c is constant and equal to 0.718 which is very close to 0.715 reported by Rouse (21) for a horizontal channel. These curves enable the discharge computation from a measured value of the end depth by making use of the following equation for rectangular weirs of finite crest width:

$$Y_c^3 = \frac{q^2}{g}$$

Discharge Equation in terms of End Depth:

Values of k in the equation $q = k\sqrt{g}\,Y_b^{3/2}$ for various values of H/P and H/B are computed for square edged weirs of finite crest width and plotted in Fig. 10.

The following equations have been obtained for the variation of k with H/P for three different values of H/B:

For H/B = 0.08, $K = 1.601 + 0.008\ H/P$

For H/B = 1.00, $K = 1.367 + 0.026\ H/P$

For H/B = 1.60, $K = 1.184 + 0.027\ H/P$

It may be noted that for $Y_b/Y_c = 0.715$, value of k is 1.654. The curve for H/B = 0.08 seems to approach the value of 1.654.

Knowing the end depth, the equation $q = k\sqrt{g}\, Y_b^{3/2}$ together with the above equations for k can be used for discharge computation.

5.7 End Depth for Weirs of Finite Crest Width with a Streamlined Upstream Corner:

The values of the ratio of the end depth to the critical depth for various values of H/B ranging from 0.05 to 0.60 and H/P ranging from 0.1 to 1.0 are presented in Table 2.

It is observed from the results shown in Table 2 that the value Y_b/Y_c is more or less constant and equal to 0.715 for values of H/B less than and equal to 0.22. It is to be noted that Rouse (21) reported a constant value of $Y_b/Y_c = 0.715$ for a horizontal smooth channel.

For values of H/B greater than 0.25 it was found that Y_b/Y_c increases indicating that these weirs do not fall into the category of broad crested ones. Therefore for H/B less than or equal to 0.22 the weirs may be classified as broad-crested weirs and the relationship $Y_b/Y_c = 0.715$ can be safely adopted. Experimental investigation for H/B between 0.22 and 0.25 was not carried out and the limiting value of H/B may be a little higher than 0.22. Within this range value of k can be taken as 1.654 and discharge computation can be made by the following equation also:

$$Q = 1.654\, L\sqrt{g}\; Y_b^{3/2}$$

CHAPTER 6

CONCLUSIONS

1. In the case of weirs width sharp upstream corner, the following equations have been found to be valid for the variation of the discharge coefficient C_d, in the equation $Q = 2/3\ C_d \sqrt{2g}\ L\ H^{3/2}$, width H/P:

For $H/B = 0.08$, $C_d = 0.482 + 0.02\ H/P$

For $H/B = 1.00$, $C_d = 0.527 + 0.049\ H/P$

For $H/B = 1.60$, $C_d = 0.578 + 0.061\ H/P$

For $H/B \geqslant 1.80$, $C_d = 0.611 + 0.080\ H/P$

Discharge may also be computed from a measured value of the end depth by making use of the Fig. 9 and the equation $Y_c^3 = q^2/g$ or from the equation $Q = k\ L \sqrt{g}\ Y_b^{3/2}$, wherein k is given by the following equations

For $H/B = 0.08$, $K = 1.601 + 0.008\ H/P$

For $H/B = 1.00$, $K = 1.367 + 0.026\ H/P$

For $H/B = 1.60$, $K = 1.184 + 0.027\ H/P$

Water surface profile was found to be consisting of standing waves for $H/B = 0.08$, parallel flow with upstream and downstream surface drawdown curves for $H/B = 1.0$ and

wholly curvelinear flow for $H/B = 1.6$, For $H/B \geq 1.8$ nappe springs clear off the crest of the weir and the weir functions as a sharp crested weir.

2. In the case of weirs with a streamlined upstream corner, for values of $H/B \leq 0.22$ and $H/P < 1.0$ up to which the investigations have been carried out, the discharge coefficient has been found to be independent of H/P and there is not much of deviation from the values given by Rouse. It has been found that the relationship $Y_b/Y_c = 0.715$ or the equation $Q = 1.654 \, L \sqrt{g} \, Y_b^{3/2}$ can be used for the computation of discharge.

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APPENDIX II - TABLE 1

DISCHARGE COEFFICIENT FOR WEIRS OF FINITE CREST WIDTH WITH A SHARP UPSTREAM CORNER FOR H/B = 1.6

Sl. No.	H ft	B in.	P in.	H/P	Q Cusecs	C_d
1	2	3	4	5	6	7
1.	0.4	3.0	1.07	4.486	1.7215	0.8480
2.	0.4	3.0	2.00	2.400	1.4919	0.7349
3.	0.4	3.0	2.5	1.920	1.3893	0.6843
4.	0.4	3.0	1.75	2.743	1.4864	0.7321
5.	0.4	3.0	1.66	2.892	1.5702	0.7734
6.	0.4	3.0	3.04	1.579	1.4125	0.6958
7.	0.4	3.0	4.25	1.130	1.3088	0.6447
8.	0.4	3.0	5.06	0.949	1.2776	0.6293
9.	0.4	3.0	6.00	0.800	1.2596	0.6204
10.	0.4	3.0	8.00	0.600	1.2329	0.6073
11.	0.4	3.0	10.12	0.474	1.1906	0.5865
12.	0.267	2.0	1.08	2.967	0.8868	0.8009
13.	0.267	2.0	2.11	1.524	0.7817	0.7021
14.	0.267	2.0	3.04	1.054	0.7226	0.6526
15.	0.267	2.0	4.25	0.754	0.6934	0.6263
16.	0.267	2.0	5.06	0.633	0.6998	0.6321
17.	0.267	2.0	8.12	0.395	0.7265	0.6562
18.	0.267	2.0	10.12	0.317	0.6779	0.6123

1	2	3	4	5	6	7
19.	0.533	4.0	2.05	3.120	2.3877	0.7646
20.	0.533	4.0	2.50	2.558	2.2423	0.7181
21.	0.533	4.0	3.05	2.097	2.1534	0.6896
22.	0.533	4.0	4.25	1.505	2.0947	0.6708
23.	0.533	4.0	5.06	1.264	2.0268	0.6490
24.	0.533	4.0	6.00	1.066	1.9946	0.6388
25.	0.133	1.0	1.10	1.451	0.2956	0.7594
26.	0.400	3.0	0.66	7.273	1.8886	0.9303
27.	0.267	2.0	0.50	6.408	1.0602	0.9576
28.	0.400	3.0	0.90	5.333	1.8561	0.9143

APPENDIX II - TABLE 2

DISCHARGE COEFFICIENT FOR WEIRS OF FINITE CREST WIDTH WITH A SHARP UPSTREAM CORNER FOR H/B = 1.0

Sl. No.	H ft	B in.	P in.	H/P	Q Cusecs	C_d
1	2	3	4	5	6	7
1.	0.333	4.0	1.08	3.700	1.1346	0.7358
2.	0.333	4.0	2.05	1.949	1.0048	0.6516
3.	0.333	4.0	3.04	1.315	0.8746	0.5671
4.	0.333	4.0	4.25	0.940	0.8541	0.5539
5.	0.333	4.0	5.06	0.790	0.8313	0.5391
6.	0.333	4.0	6.00	0.660	0.8162	0.5293
7.	0.333	4.0	2.50	1.598	0.8948	0.5803
8.	0.25	3.0	1.07	2.804	0.7067	0.7045
9.	0.25	3.0	2.0	1.500	0.6022	0.6003
10.	0.25	3.0	2.5	1.200	0.5978	0.5960
11.	0.25	3.0	1.75	1.714	0.6196	0.6177
12.	0.25	3.0	1.65	1.818	0.6234	0.6214
13.	0.25	3.0	0.75	4.00	0.7339	0.7316
14.	0.25	3.0	0.66	4.545	0.7658	0.7634
15.	0.25	3.0	3.04	0.987	0.6003	0.5985
16.	0.25	3.0	4.25	0.706	0.5895	0.5877
17.	0.25	3.0	5.06	0.593	0.5826	0.5808
18	0.25	3.0	6.0	0.500	0.5597	0.5580

1	2	3	4	5	6	7
19.	0.25	3.0	8.0	0.375	0.5371	0.5354
20.	0.25	3.0	10.12	0.296	0.5275	0.5258
21.	0.167	2.0	2.11	0.950	0.3125	0.5706
22.	0.167	2.0	1.08	1.856	0.3249	0.5932
23.	0.167	2.0	3.04	0.659	0.3146	0.5744
24.	0.167	2.0	4.25	0.472	0.3090	0.5642
25.	0.167	2.0	10.12	0.198	0.2955	0.5396
26.	0.167	2.0	6.12	0.328	0.3246	0.5927
27.	0.167	2.0	5.06	0.396	0.2729	0.4983
28.	0.416	5.0	1.05	4.754	1.6219	0.7533
29.	0.416	5.0	2.06	2.423	1.4463	0.6717
30.	0.333	4.0	0.66	6.055	1.2659	0.8209

APPENDIX II - TABLE 3

DISCHARGE COEFFICIENT FOR WEIRS OF FINITE CREST WIDTH WITH A SHARP UPSTREAM CORNER FOR H/B = 0.08

Sl. No.	H ft	B in.	P in.	H/P	Q Cusecs	C_d
1.	0.333	50.0	5.08	0.787	0.7628	0.4947
2.	0.333	50.0	4.03	0.992	0.8068	0.5232
3.	0.333	50.0	3.00	1.332	0.7722	0.5008
4.	0.333	50.0	2.03	1.969	0.8406	0.5451
5.	0.200	30.0	2.04	1.177	0.3669	0.5112
6.	0.266	40.0	1.13	2.825	0.5997	0.5447
7.	0.200	30.0	1.02	2.353	0.3469	0.4833
8.	0.200	30.0	1.50	1.600	0.3706	0.5163
9.	0.333	50.0	1.50	2.664	0.8402	0.5448
10.	0.333	50.0	1.05	3.806	0.8531	0.5532
11.	0.400	60.0	1.00	4.800	0.7686	0.5700
12.	0.400	60.0	0.80	6.000	0.7985	0.5900

APPENDIX II - TABLE 4

END DEPTH FOR WEIRS OF FINITE CREST WIDTH WITH A SHARP UPSTREAM CORNER FOR H/B = 1.6

Sl. No.	H/P	Y_b ft	Y_c ft	Y_b/Y_c	k
1	2	3	4	5	6
1.	4.4860	0.2890	0.3449	0.8379	1.3018
2.	2.4000	0.2890	0.3136	0.9217	1.1282
3.	1.9200	0.2620	0.2990	0.8762	1.2171
4.	2.7429	0.2660	0.3128	0.8504	1.2729
5.	2.8916	0.2720	0.3244	0.8384	1.3004
6.	1.5789	0.2510	0.3023	0.8302	1.3197
7.	1.1294	0.2530	0.2874	0.8804	1.2083
8.	0.9486	0.2480	0.2828	0.8770	1.2154
9.	0.8000	0.2490	0.2801	0.8889	1.1910
10.	0.6000	0.2430	0.2762	0.8799	1.2092
11.	0.4773	0.2320	0.2698	0.8598	1.2517
12.	2.9667	0.1950	0.2217	0.8794	1.2098
13.	1.5242	0.1750	0.2039	0.8583	1.2546
14.	1.0539	0.1720	0.1935	0.8890	1.1900
15.	0.7539	0.1640	0.1882	0.8712	1.2266
16.	0.6332	0.1650	0.1894	0.8712	1.2267
17.	0.3946	0.1670	0.1942	0.8601	1.2506
18.	0.3166	0.1680	0.1854	0.9061	1.1565

1	2	3	4	5	6
19.	3.1200	0.3830	0.4289	0.8930	1.1835
20.	2.5584	0.3580	0.4113	0.8704	1.2299
21.	2.0970	0.3540	0.4004	0.8842	1.2012
22.	1.5049	0.3440	0.3931	0.8752	1.2197
23.	1.2640	0.3390	0.3845	0.8816	1.2064
24.	1.0660	0.3300	0.3805	0.8674	1.2362
25.	1.4509	0.1050	0.1067	0.9842	1.0207
26.	7.2727	0.2870	0.3669	0.7823	1.4431
27.	6.4080	0.2010	0.2498	0.8048	1.3822
28.	5.3333	0.3060	0.3627	0.8438	1.2882

APPENDIX II - TABLE 5

END DEPTH FOR WEIRS OF FINITE CREST WIDTH WITH A SHARP UPSTREAM CORNER FOR H/B = 1.0

Sl. No.	H/P	Y_b ft	Y_c ft	Y_b/Y_c	k
1	2	3	4	5	6
1.	3.7000	0.2060	0.2613	0.7884	1.4257
2.	1.9493	0.1880	0.2410	0.7801	1.4482
3.	1.3145	0.1630	0.2197	0.7419	1.5613
4.	0.9402	0.1660	0.2163	0.7676	1.4837
5.	0.7897	0.1660	0.2124	0.7815	1.4441
6.	0.6660	0.1670	0.2098	0.7959	1.4051
7.	1.5984	0.1780	0.2231	0.7979	1.3999
8.	2.8037	0.1480	0.1906	0.7767	1.4582
9.	1.5000	0.1340	0.1714	0.7820	1.4423
10.	1.2000	0.1350	0.1705	0.7917	1.4159
11.	1.7143	0.1410	0.1746	0.8074	1.3749
12.	1.8182	0.1380	0.1753	0.7870	1.4286
13.	4.000	0.1500	0.1955	0.7673	1.4841
14.	4.5455	0.1520	0.2011	0.7558	1.5183
15.	0.9868	0.1380	0.1710	0.8070	1.3758
16.	0.7059	0.1260	0.1689	0.7458	1.5485
17.	0.5929	0.1350	0.1676	0.8054	1.3799
18.	0.5000	0.1320	0.1632	0.8087	1.3712

1	2	3	4	5	6
19.	0.3750	0.1290	0.1588	0.8124	1.3620
20.	0.2964	0.1310	0.1569	0.8350	1.3070
21.	0.9498	0.0780	0.1107	0.7046	1.6853
22.	1.8556	0.0980	0.1136	0.8626	1.2441
23.	0.6592	0.0950	0.1112	0.8543	1.2623
24.	0.4715	0.0880	0.1099	0.8009	1.3906
25.	0.1980	0.0950	0.1067	0.8907	1.1857
26.	0.3275	0.0950	0.1135	0.8367	1.3023
27.	0.3960	0.0880	0.1012	0.8700	1.2281
28.	4.7543	0.2540	0.3315	0.7662	1.4885
29.	2.4233	0.2400	0.3071	0.7814	1.4452
30.	6.0545	0.2180	0.2811	0.7756	1.4611

APPENDIX II - TABLE 6

END DEPTH FOR WEIRS OF FINITE CREST WIDTH WITH A SHARP UPSTREAM CORNER FOR H/B - 0.08

Sl. No.	H/P	Y_b ft	Y_c ft	Y_b/Y_c	k
1.	0.7866	0.145	0.2006	0.7229	1.6231
2.	0.9916	0.157	0.2082	0.7540	1.5236
3.	0.3320	0.146	0.2022	0.7220	1.6262
4.	1.9685	0.164	0.2140	0.7664	1.4870
5.	1.1765	0.088	0.1232	0.7143	1.6513
6.	2.8248	0.123	0.1709	0.7198	1.6333
7.	2.3529	0.080	0.1187	0.7246	1.6161
8.	1.6000	0.087	0.1240	0.7015	1.6965
9.	2.6640	0.150	0.2139	0.7012	1.6991
10.	3.8057	0.156	0.2161	0.7219	1.6266
11.	4.8000	0.146	0.2016	0.7242	1.6187
12.	6.0000	0.148	0.2068	0.7157	1.6477

APPENDIX II - TABLE 7

L=18", DISCHARGE COEFFICIENTS FOR STREAMLINED WEIRS OF FINITE CREST WIDTH

Sl. No.	H ft	B in.	P in.	H/B	H/P	L/B	Q Cusecs	C_d	$C = \frac{2}{3}\frac{C_d\sqrt{2g}}{3.09}$	C Rouse(7)	% deviation
1	2	3	4	5	6	7	8	9	10	11	12
1.	0.125	30.0	10.08	0.05	0.149	0.60	0.2009	0.5664	0.9810	0.891	+10.00
2.	0.250	60.0	15.08	0.05	0.200	0.30	0.5272	0.5255	0.9120	0.915	-00.30
3.	0.250	60.0	6.00	0.05	0.500	0.30	0.5534	0.5516	0.9550	0.915	+ 4.38
4.	0.246	59.0	5.14	0.05	0.94	0.305	0.5137	0.5247	0.9090	0.950	- 4.30
5.	0.167	20.0	10.20	0.10	0.197	0.90	0.3007	0.5490	0.9530	0.940	+ 1.38
6.	0.25	30.0	10.08	0.10	0.298	0.60	0.5400	0.5380	0;9340	0.944	- 1:06
7.	0.283	34.0	5.10	0.10	0.666	0.53	0.6554	0.5425	0.9400	0.945	- 0.53
8.	0.408	49.0	5.10	0.10	0.96	0.37	1.1610	0.5552	0.9640	0.952	+ 1.26
9.	0.333	40.0	4.0	0.10	1.0	0.45	0.8433	0.5469	0.9490	0.948	0.00
10.	0.250	20.0	10.2	0.15	0.294	0.90	0.5591	0.5573	0.9660	0.958	+ 0.84
11.	0.375	30.0	10.08	0.15	0.446	0.60	1.0026	0.5441	0.9430	0.961	- 2.06
12.	0.425	34.0	5.1	0.15	1.0	0.37	1.2230	0.5500	0.9540	0.962	- 0.84
13.	0.300	20.0	10.2	0.16	0.353	0.90	0.7366	0.5586	0.9675	-	-
14.	0.333	20.0	10.2	0.2	0.392	0.90	0.8441	0.5474	0.9500	-	-

1	2	3	4	5	6	7	8	9	10	11	12
15.	0.367	20.0	10.2	0.22	0.432	0.90	0.9794	0.5489	0.9518	-	-
16.	0.5000	30.0	10.08	0.20	0.60	0.60	1.6387	0.5776	1.0050	-	-
17.	0.416	20.0	10.2	0.25	0.49	0.90	1.2071	0.5606	0.9705	-	-
18.	0.625	30.0	10.08	0.25	0.74	0.60	2.2539	0.5684	0.9855	-	-
19.	0.500	20.0	10.20	0.30	0.59	0.90	1.6158	0.5695	0.9875	-	-
20.	0.500	10.0	10.00	0.60	0.60	1.80	1.8492	0.6517	1.1300	-	-
21.	0.333	10.0	10.00	0.40	0.40	1.80	0.9910	0.6427	1.1150	-	-
22.	0.416	10.0	10.00	0.50	0.50	1.80	1.3978	0.6492	1.1270	-	-
23.	0.250	10.0	10.00	0.30	0.30	1.80	0.6542	0.6522	1.1320	-	-
24.	0.208	10.0	10.00	0.25	0.25	1.80	0.5026	0.6602	1.1440	-	-
25.	0.167	10.0	10.00	0.20	0.20	1.80	0.2981	0.5443	0.9430	-	-

APPENDIX II - TABLE 8

RATIO OF THE END DEPTH TO THE CRITICAL DEPTH FOR STREAMLINED WEIRS OF FINITE CREST WIDTH

Sl. No.	H/B	H/P	C_d	Y_c ft	Y_b ft	$\frac{Y_b}{Y_c}$	K
1	2	3	4	5	6	7	8
1.	0.05	0.1488	0.5664	0.0825	0.0590	0.7153	1.6468
2.	0.05	0.1989	0.5255	0.1568	0.1120	0.7142	1.6524
3.	0.05	0.5000	0.5510	0.1620	0.1150	0.7100	1.6670
4.	0.05	0.9401	0.5247	0.1542	0.110	0.7136	1.6543
5.	0.10	0.1965	0.5490	0.1079	0.077	0.7136	1.6533
6.	0.10	0.2976	0.9589	0.1594	0.1140	0.7156	1.6483
7.	0.10	0.6659	0.5425	0.1813	0.1290	0.7115	1.6619
8.	0.10	0.9600	0.5552	0.2653	0.1900	0.7161	1.6470
9.	0.10	0.9990	0.5469	0.2144	0.153	0.7135	1.6556
10.	0.15	0.2941	0.5573	0.1631	0.117	0.7174	1.6412
11.	0.15	0.4464	0.5441	0.2406	0.173	0.7189	1.6370
12.	0.15	1.000	0.5500	0.2747	0.197	0.7172	1.6433
13.	0.18	0.3529	0.5586	0.1960	0.138	0.7042	1.6880
14.	0.20	0.3918	0.5474	0.2146	0.153	0.7130	1.6570
15.	0.22	0.4318	0.5489	0.2369	0.170	0.7176	1.6416
16.	0.20	0.5952	0.5776	0.3338	0.240	0.7190	1.6375
17.	0.25	0.4894	0.5606	0.2723	0.198	0.7272	1.6096
18.	0.25	0.7440	0.5684	0.4127	0.301	0.7293	1.6035

1	2	3	4	5	6	7	8
19.	0.25	0.7440	0.5684	0.4127	0.301	0.7293	1.6035
20.	0.60	0.6000	0.6517	0.3618	0.309	0.8542	1.2648
21.	0.40	0.4000	0.6427	0.2388	0.197	0.8250	1.3316
22.	0.50	0.5000	0.6492	0.3002	0.255	0.8493	1.2753
23.	0.30	0.3000	0.6522	0.1811	0.142	0.7842	1.4363
24.	0.25	0.2500	0.6602	0.1519	0.115	0.7569	1.5142
25.	0.20	0.2000	0.5443	0.1073	0.078	0.7271	1.6076

APPENDIX III

NOTATION

A_c = Area of the critical section

B = With of the weir

b = Contracted width of the measuring notch

b_e = Effective contracted width of the notch

C = Coefficient in the equation $q = 3.09\ C\ H^{3/2}$

C_d = Discharge coefficient

C_e = Coefficient in the equation $Q = C_e\ b_e\ h_e^{3/2}$

C_c = Coefficient of contraction

g = Acceleration due to gravity

H = Piezometric head over the weir

H_O = Average energy head referred to crest level

h = Head over the measuring notch

k = Coefficient in the equation $q = k\sqrt{g}\ Y_b^{3/2}$

L = Length of the weir

P = Height of the weir

q = Discharge per unit width

Q = Discharge

T = Width of the water surface

V_O = Velocity of approach

Y_b = End depth or brink depth

Y_c = Hydrostatic critical depth

γ = Specific weight of the fluid

ρ = Density of the fluid

μ = Dynamic viscosity of the fluid

σ = Surface tension of the fluid

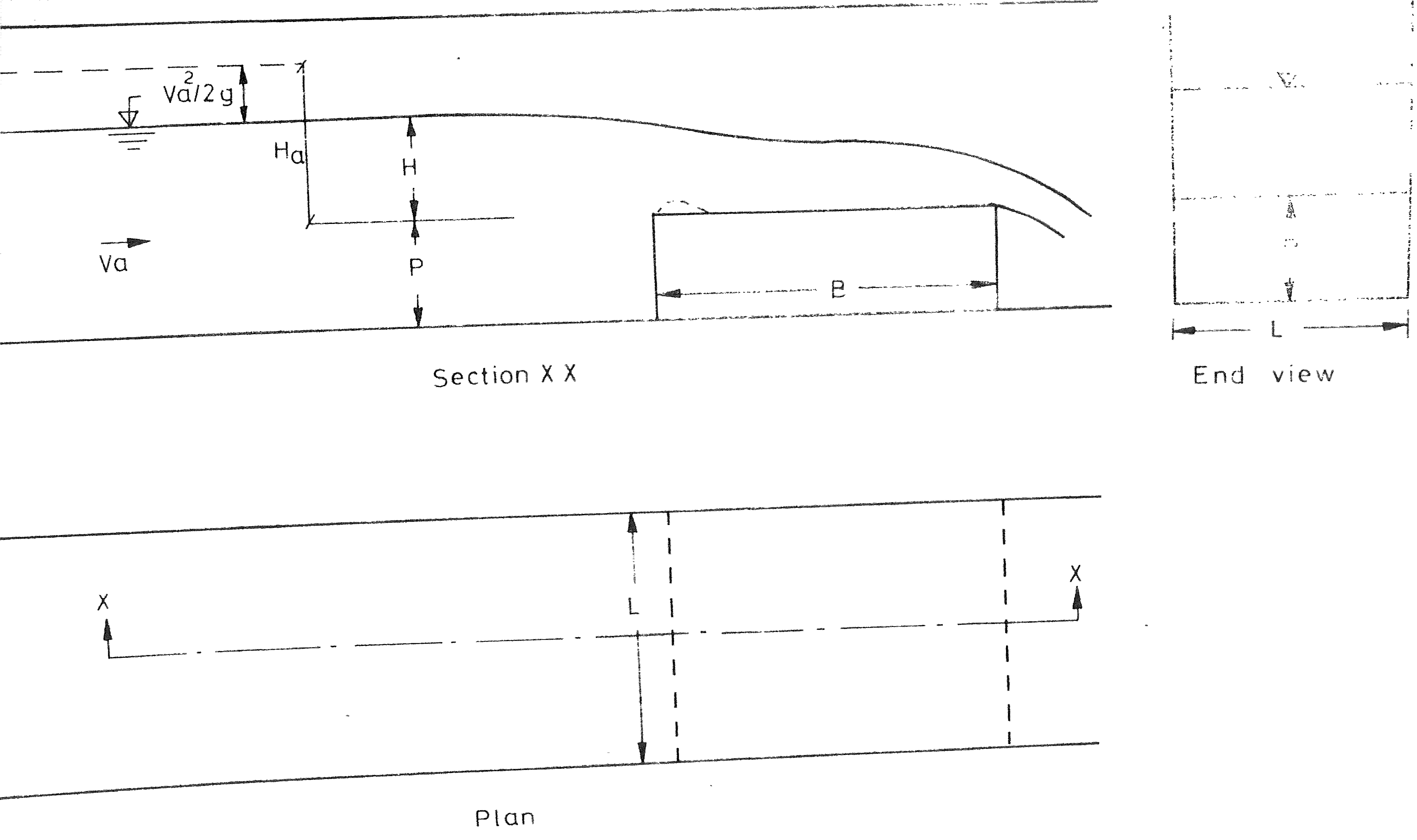

FIG. 1: Definition Sketch-Flow over a Full Width Weir with Confined Nappe.

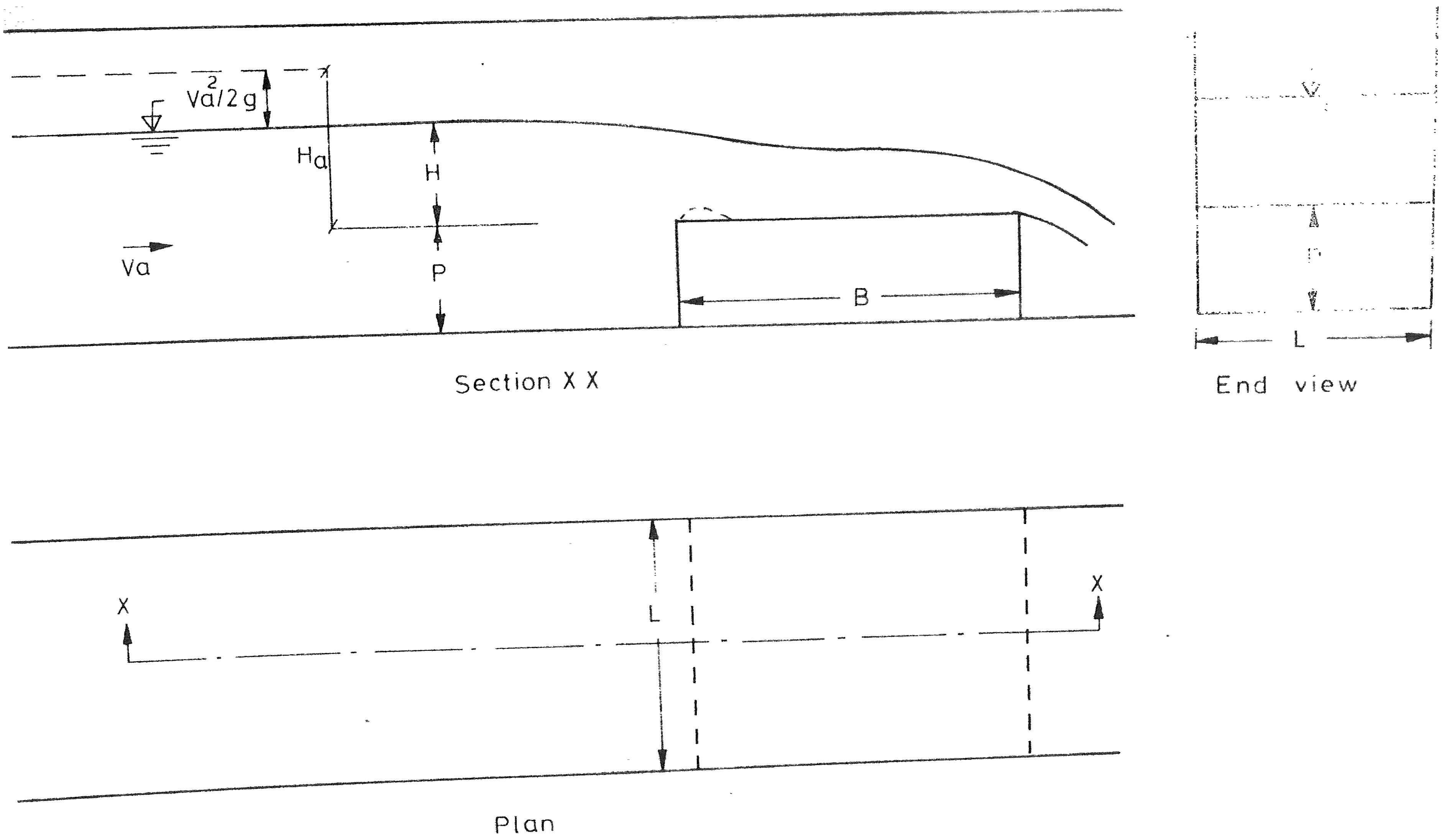

FIG. 1: Definition Sketch-Flow over a Full Width Weir with Confined Nappe.

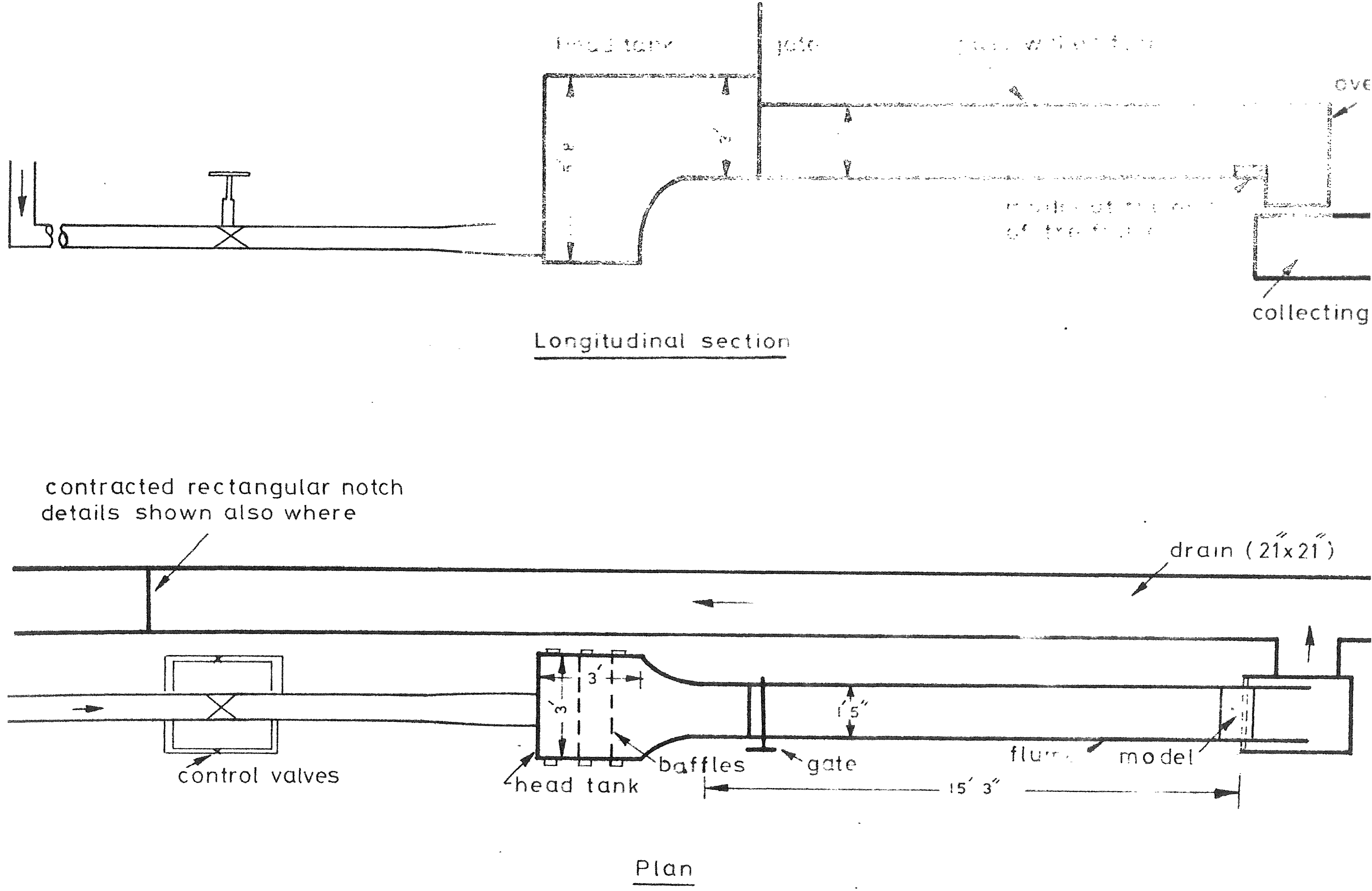

FIG. 2: Schematic View of the Experimental Equipment.

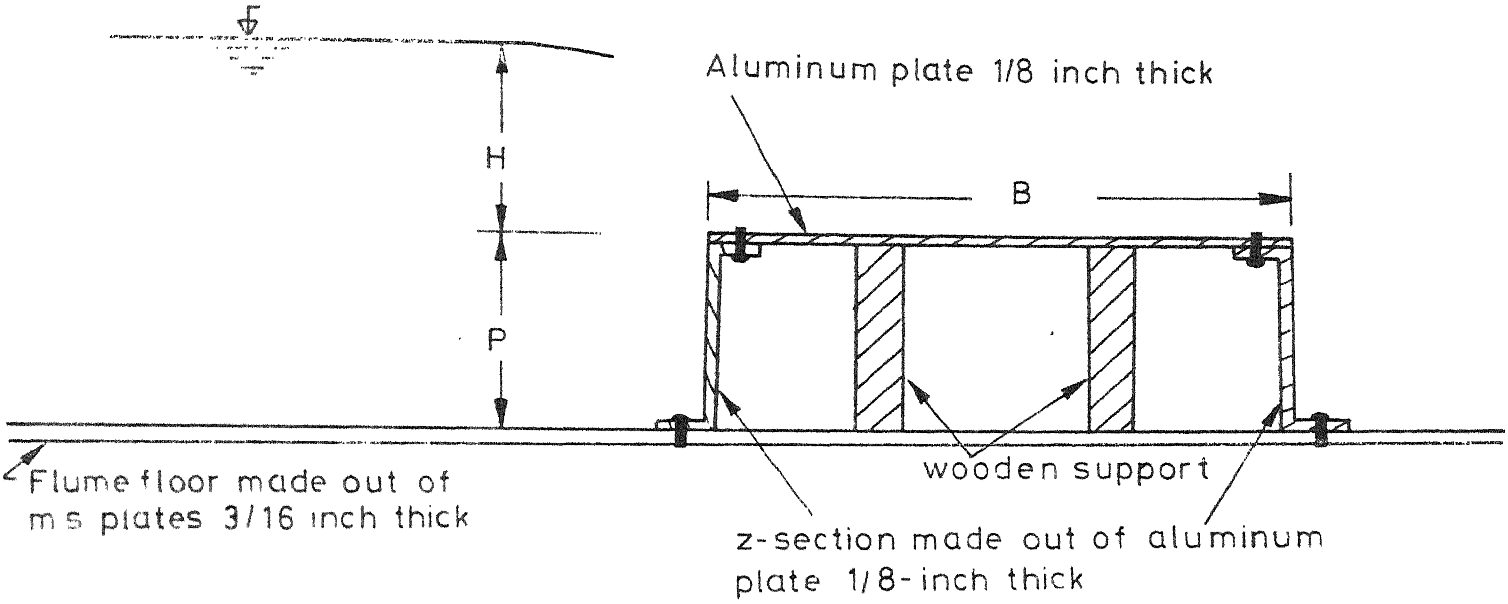

FIG.3 Elevation of a typical weir assembly installed in the flume

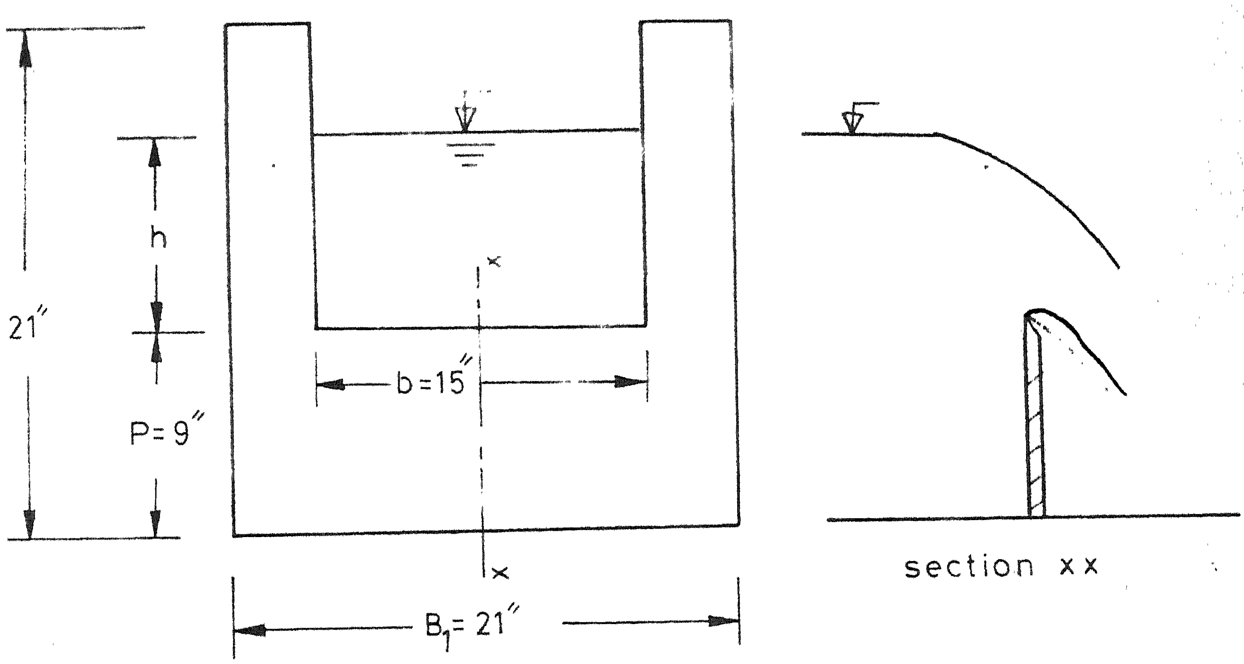

FIG. 4: Rectangular Contracted Sharp-Crested Weir for Flow Measurement.

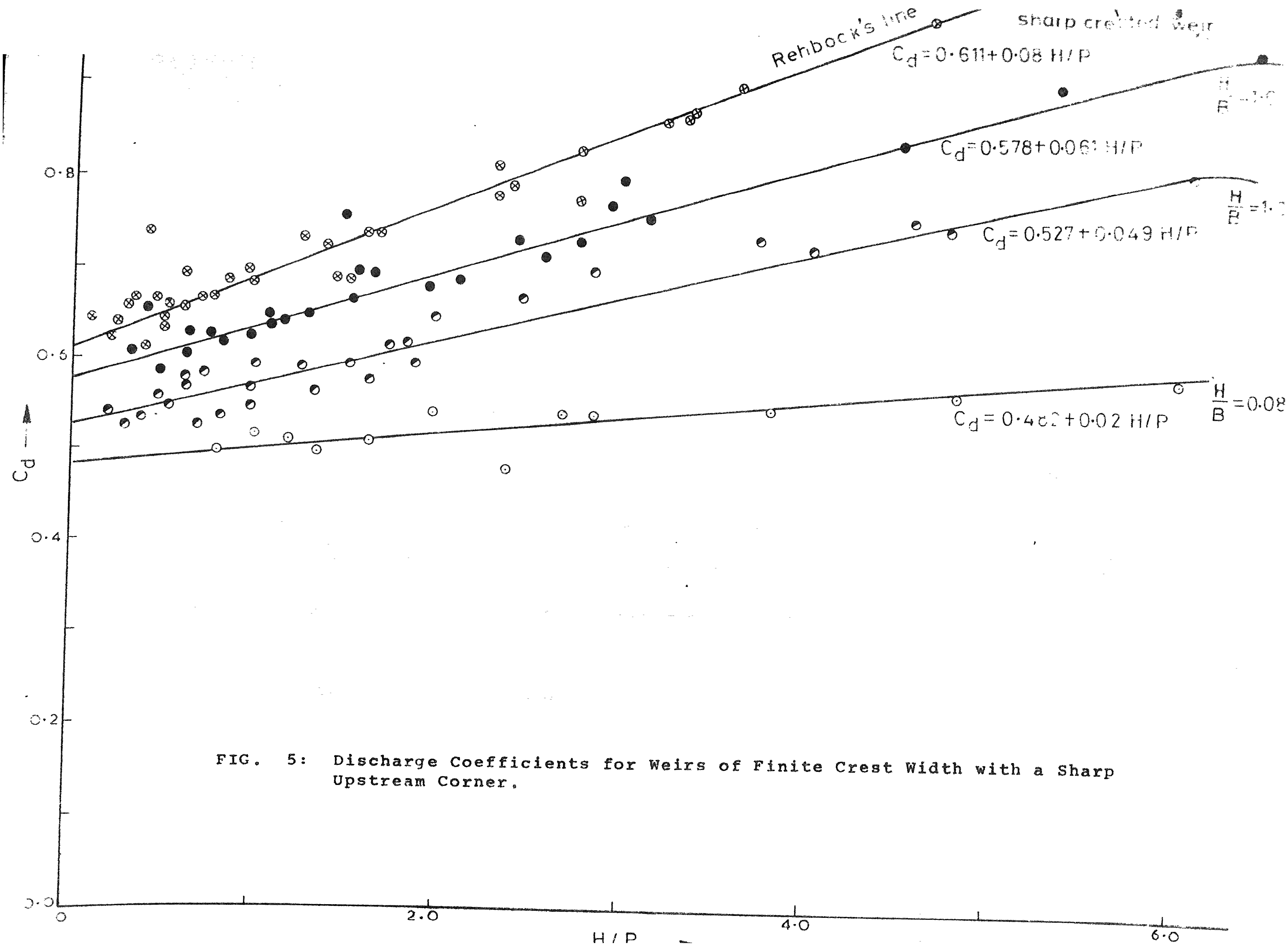

FIG. 5: Discharge Coefficients for Weirs of Finite Crest Width with a Sharp Upstream Corner.

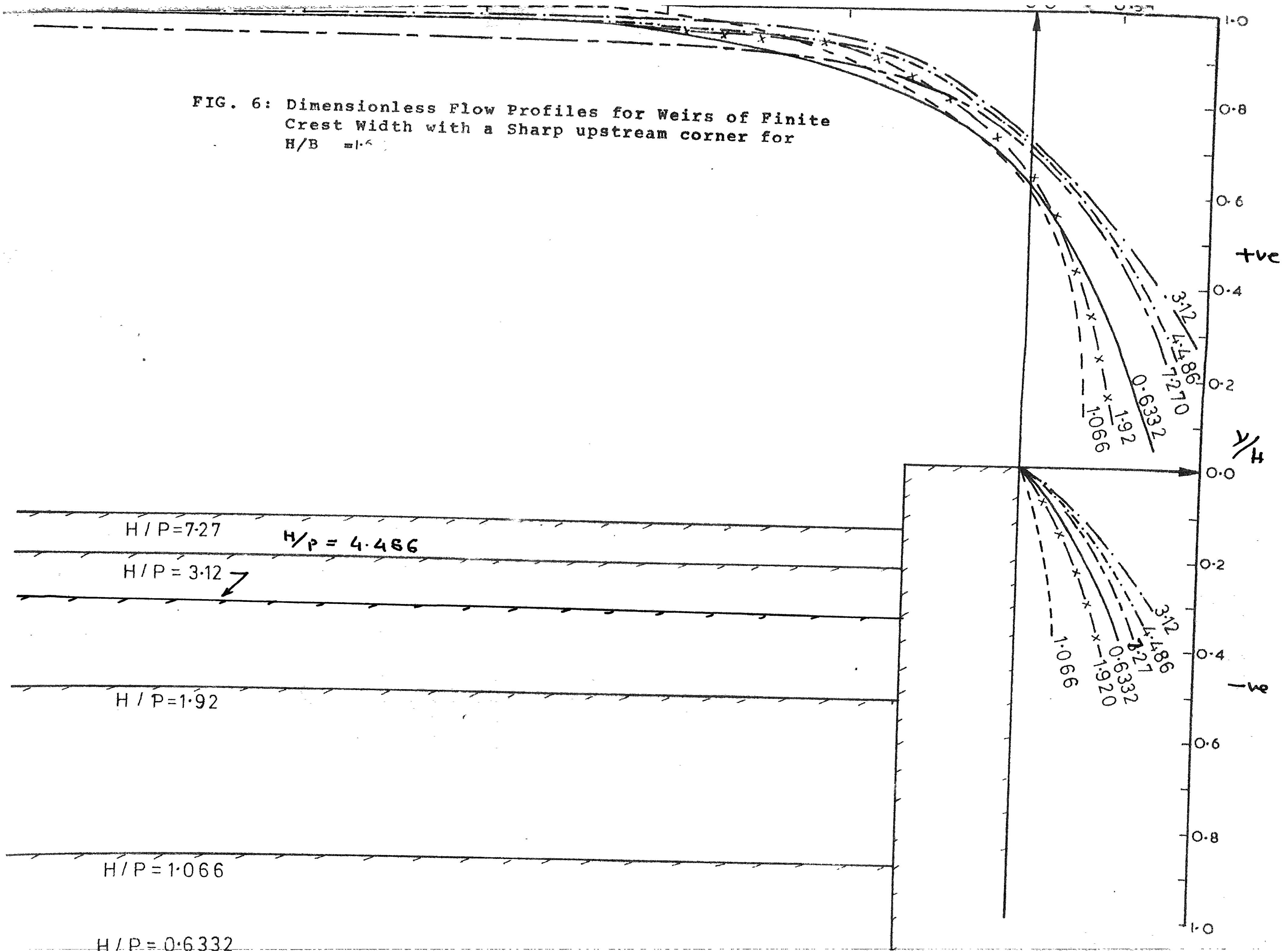

FIG. 6: Dimensionless Flow Profiles for Weirs of Finite Crest Width with a Sharp upstream corner for H/B =1.6

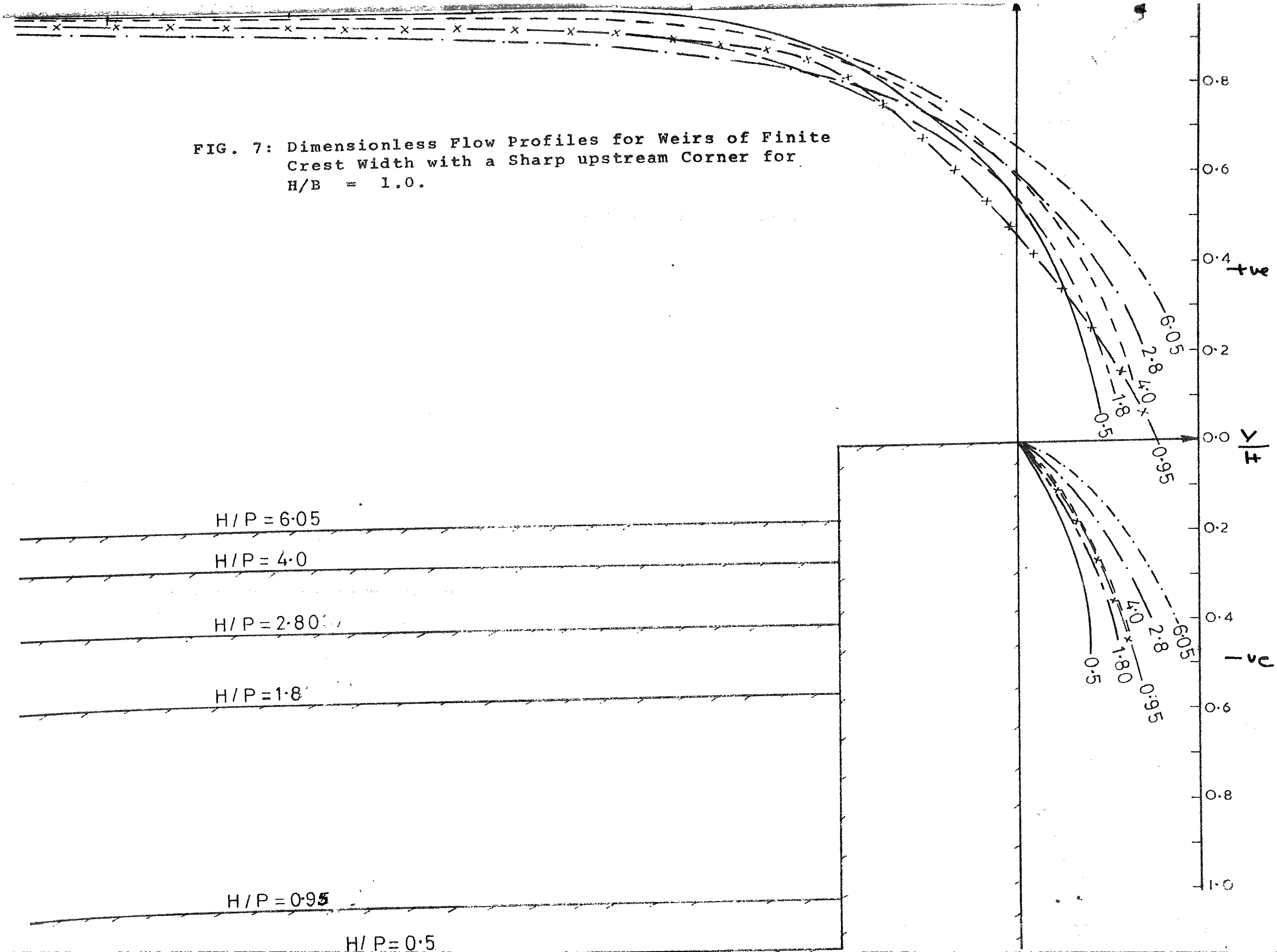

FIG. 7: Dimensionless Flow Profiles for Weirs of Finite Crest Width with a Sharp upstream Corner for H/B = 1.0.

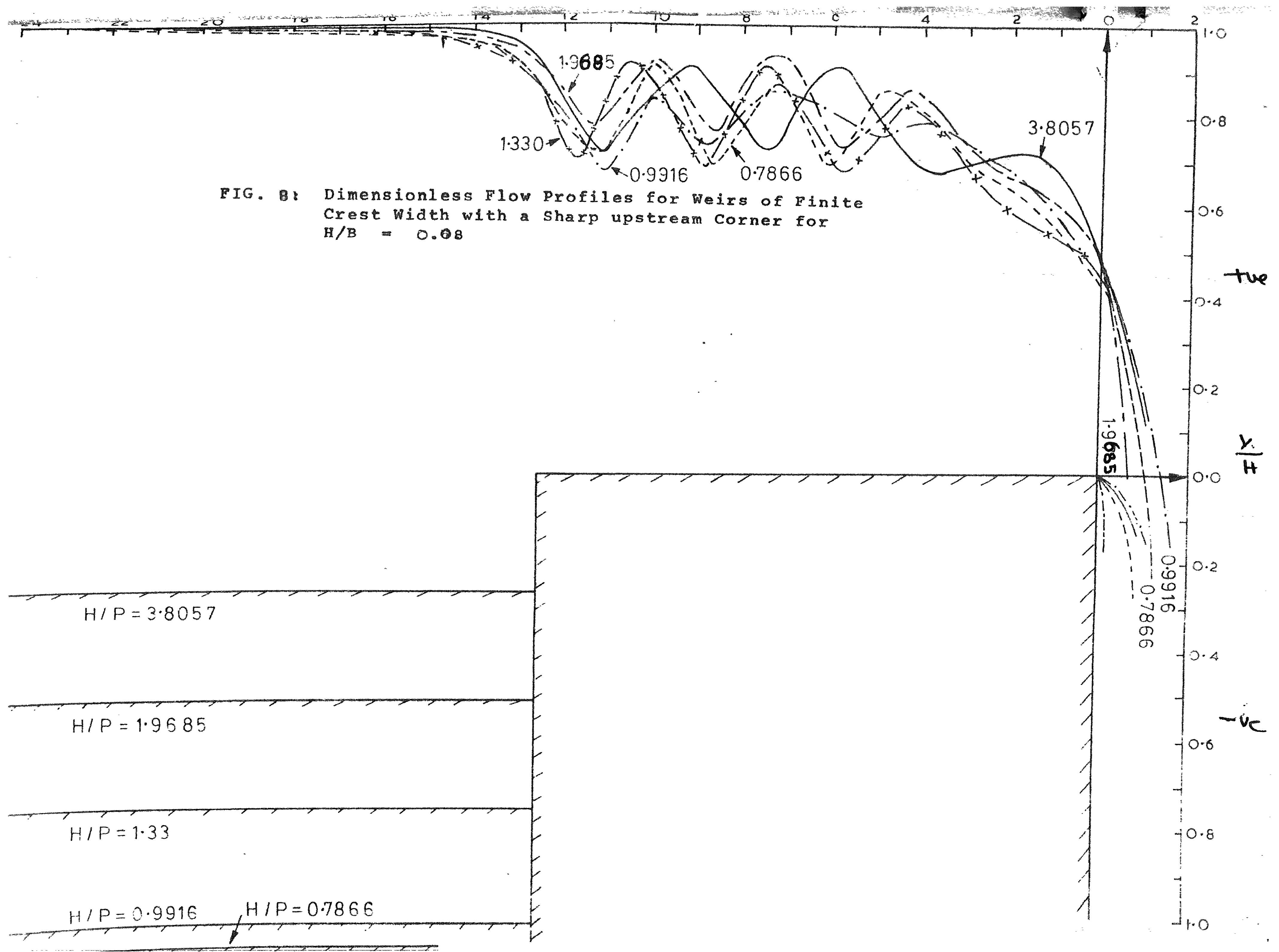

FIG. 8: Dimensionless Flow Profiles for Weirs of Finite Crest Width with a Sharp upstream Corner for H/B = 0.08

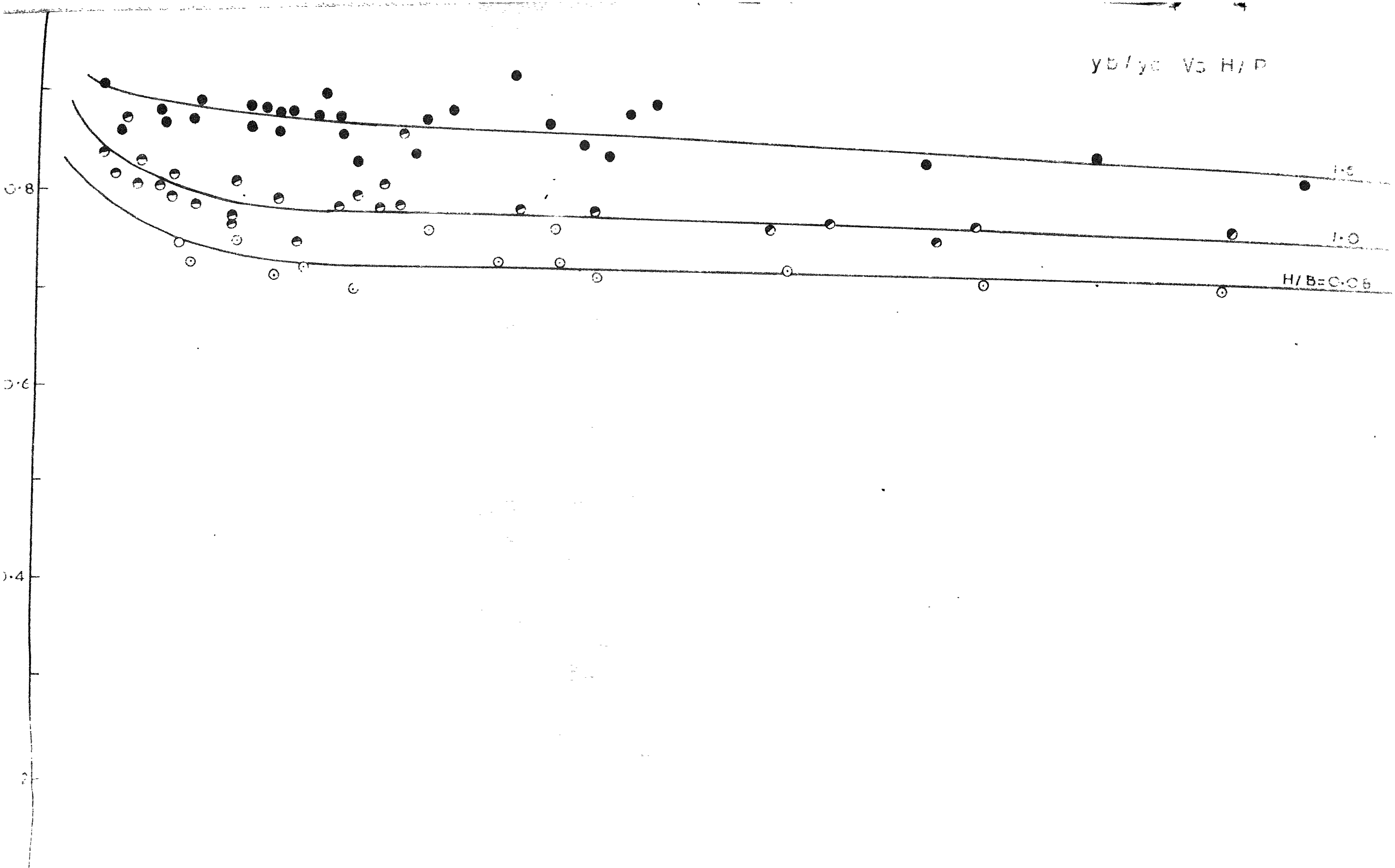

FIG. 9: Values of the Ratio of the End Dephh to the Critical Depth for Weirs of Finite Crest Width with a Sharp upstream Corner.

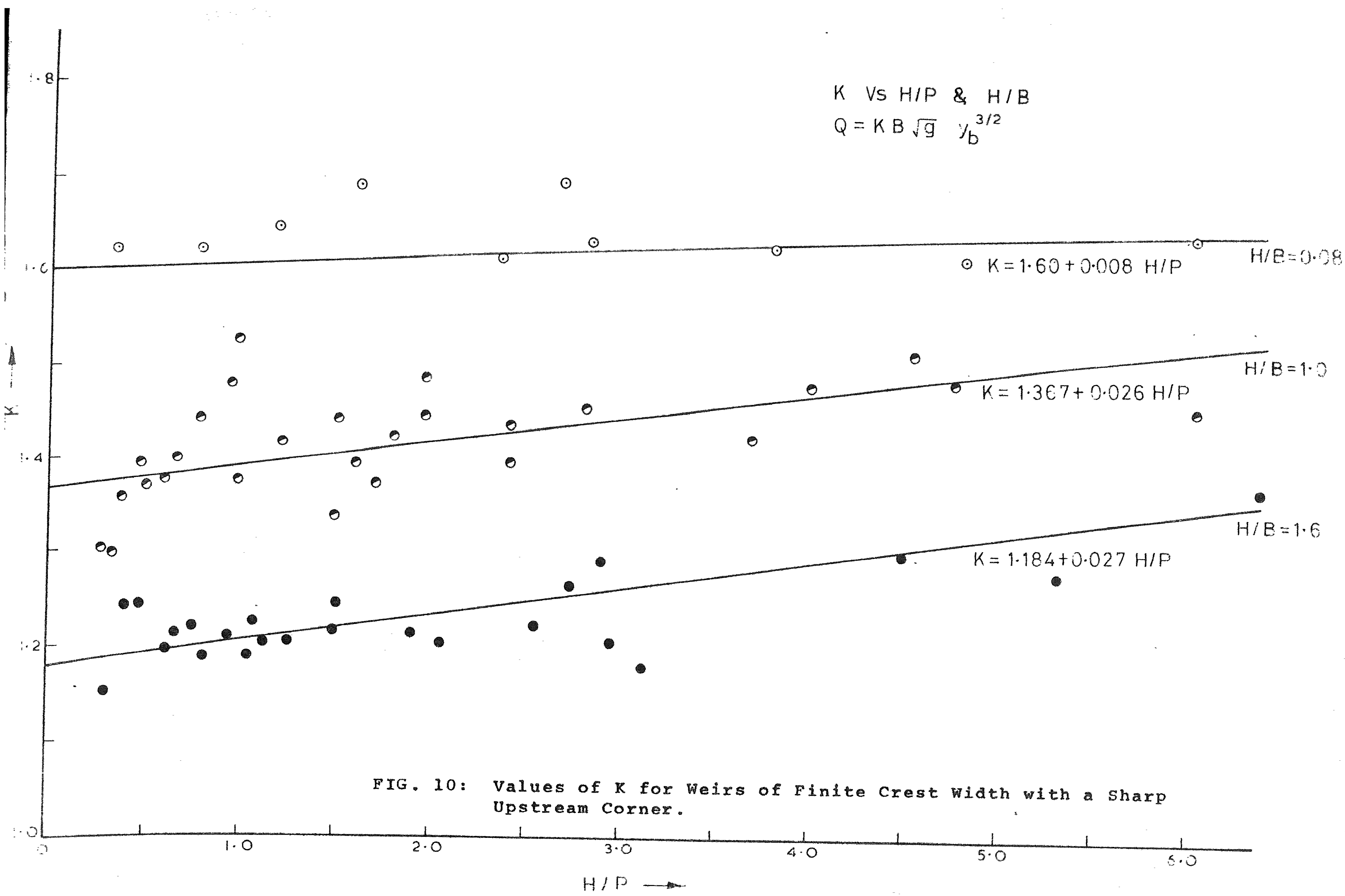

FIG. 10: Values of K for Weirs of Finite Crest Width with a Sharp Upstream Corner.